Chapter 3
Design Data

3-1. Concrete Properties

a. General. The specific concrete properties used in the design of concrete gravity dams include the unit weight, compressive, tensile, and shear strengths, modulus of elasticity, creep, Poisson's ratio, coefficient of thermal expansion, thermal conductivity, specific heat, and diffusivity. These same properties are also important in the design of RCC dams. Investigations have generally indicated RCC will exhibit properties equivalent to those of conventional concrete. Values of the above properties that are to be used by the designer in the reconnaissance and feasibility design phases of the project are available in ACI 207.1R-87 or other existing sources of information on similar materials. Follow-on laboratory testing and field investigations should provide the values necessary in the final design. Temperature control and mix design are covered in EM 1110-2-2000 and Em 1110-2-2006.

b. Strength.

(1) Concrete strength varies with age; the type of cement, aggregates, and other ingredients used; and their proportions in the mixture. The main factor affecting concrete strength is the water-cement ratio. Lowering the ratio improves the strength and overall quality. Requirements for workability during placement, durability, minimum temperature rise, and overall economy may govern the concrete mix proportioning. Concrete strengths should satisfy the early load and construction requirements and the stress criteria described in Chapter 4. Design compressive strengths at later ages are useful in taking full advantage of the strength properties of the cementitious materials and lowering the cement content, resulting in lower ultimate internal temperature and lower potential cracking incidence. The age at which ultimate strength is required needs to be carefully reviewed and revised where appropriate.

(2) Compressive strengths are determined from the standard unconfined compression test excluding creep effects (American Society for Testing and Materials (ASTM) C 39, "Test Method for Compressive Strength of Cylindrical Concrete Specimens"; C 172, "Method of Sampling Freshly Mixed Concrete"; ASTM C 31, "Method of Making and Curing Concrete Test Specimens in the Field").

(3) The shear strength along construction joints or at the interface with the rock foundation can be determined by the linear relationship $T = C + \delta \tan \phi$ in which C is the unit cohesive strength, δ is the normal stress, and $\tan \phi$ represents the coefficient of internal friction.

(4) The splitting tension test (ASTM C 496) or the modulus of rupture test (ASTM C 78) can be used to determine the strength of intact concrete. Modulus of rupture tests provide results which are consistent with the assumed linear elastic behavior used in design. Spitting tension test results can be used; however, the designer should be aware that the results represent nonlinear performance of the sample. A more detailed discussion of these tests is presented in the *ACI Journal* (Raphael 1984).

c. Elastic properties.

(1) The graphical stress-strain relationship for concrete subjected to a continuously increasing load is a curved line. For practical purposes, however, the modulus of elasticity is considered a constant for the range of stresses to which mass concrete is usually subjected.

(2) The modulus of elasticity and Poisson's ratio are determined by the ASTM C 469, "Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression."

(3) The deformation response of a concrete dam subjected to sustained stress can be divided into two parts. The first, elastic deformation, is the strain measured immediately after loading and is expressed as the instantaneous modulus of elasticity. The other, a gradual yielding over a long period, is the inelastic deformation or creep in concrete. Approximate values for creep are generally based on reduced values of the instantaneous modulus. When design requires more exact values, creep should be based on the standard test for creep of concrete in compression (ASTM C 512).

d. Thermal properties. Thermal studies are required for gravity dams to assess the effects of stresses induced by temperature changes in the concrete and to determine the temperature controls necessary to avoid undesirable cracking. The thermal properties required in the study include thermal conductivity, thermal diffusivity, specific heat, and the coefficient of thermal expansion.

e. Dynamic properties.

(1) The concrete properties required for input into a linear elastic dynamic analysis are the unit weight, Young's modulus of elasticity, and Poisson's ratio. The

concrete tested should be of sufficient age to represent the ultimate concrete properties as nearly as practicable. One-year-old specimens are preferred. Usually, upper and lower bound values of Young's modulus of elasticity will be required to bracket the possibilities.

(2) The concrete properties needed to evaluate the results of the dynamic analysis are the compressive and tensile strengths. The standard compression test (see paragraph 3-1*b*) is acceptable, even though it does not account for the rate of loading, since compression normally does not control in the dynamic analysis. The splitting tensile test or the modulus of rupture test can be used to determine the tensile strength. The static tensile strength determined by the splitting tensile test may be increased by 1.33 to be comparable to the standard modulus of rupture test.

(3) The value determined by the modulus of rupture test should be used as the tensile strength in the linear finite element analysis to determine crack initiation within the mass concrete. The tensile strength should be increased by 50 percent when used with seismic loading to account for rapid loading. When the tensile stress in existing dams exceeds 150 percent of the modulus of rupture, nonlinear analyses will be required in consultation with CECW-ED to evaluate the extent of cracking. For initial design investigations, the modulus of rupture can be calculated from the following equation (Raphael 1984):

$$f_t = 2.3 f_c'^{\,2/3} \tag{3-1}$$

where

f_t = tensile strength, psi (modulus of rupture)

f_c' = compressive strength, psi

3-2. Foundation Properties

a. Deformation modulus. The deformation modulus of a foundation rock mass must be determined to evaluate the amount of expected settlement of the structure placed on it. Determination of the deformation modulus requires coordination of geologists and geotechnical and structural engineers. The deformation modulus may be determined by several different methods or approaches, but the effect of rock inhomogeneity (due partially to rock discontinuities) on foundation behavior must be accounted for. Thus, the determination of foundation compressibility should consider both elastic and inelastic (plastic) deformations. The resulting "modulus of deformation" is a lower value than the elastic modulus of intact rock. Methods for evaluating foundation moduli include in situ (static) testing (plate load tests, dilatometers, etc.); laboratory testing (uniaxial compression tests, ASTM C 3148; and pulse velocity test, ASTM C 2848); seismic field testing; empirical data (rock mass rating system, correlations with unconfined compressive strength, and tables of typical values); and back calculations using compression measurements from instruments such as a borehole extensometer. The foundation deformation modulus is best estimated or evaluated by in situ testing to more accurately account for the natural rock discontinuities. Laboratory testing on intact specimens will yield only an "upper bound" modulus value. If the foundation contains more than one rock type, different modulus values may need to be used and the foundation evaluated as a composite of two or more layers.

b. Static strength properties. The most important foundation strength properties needed for design of concrete gravity structures are compressive strength and shear strength. Allowable bearing capacity for a structure is often selected as a fraction of the average foundation rock compressive strength to account for inherent planes of weakness along natural joints and fractures. Most rock types have adequate bearing capacity for large concrete structures unless they are soft sedimentary rock types such as mudstones, clayshale, etc.; are deeply weathered; contain large voids; or have wide fault zones. Foundation rock shear strength is given as two values: cohesion (c) and internal friction (ϕ). Design values for shear strength are generally selected on the basis of laboratory direct shear test results. Compressive strength and tensile strength tests are often necessary to develop the appropriate failure envelope during laboratory testing. Shear strength along the foundation rock/structure interface must also be evaluated. Direct shear strength laboratory tests on composite grout/rock samples are recommended to assess the foundation rock/structure interface shear strength. It is particularly important to determine strength properties of discontinuities and the weakest foundation materials (i.e., soft zones in shears or faults), as these will generally control foundation behavior.

c. Dynamic strength properties.

(1) When the foundation is included in the seismic analysis, elastic moduli and Poisson's ratios for the foundation materials are required for the analysis. If the foundation mass is modeled, the rock densities are also required.

(2) Determining the elastic moduli for a rock foundation should include several different methods or approaches, as defined in paragraph 3-2*a*.

(3) Poisson's ratios should be determined from uniaxial compression tests, pulse velocity tests, seismic field tests, or empirical data. Poisson's ratio does not vary widely for rock materials.

(4) The rate of loading effect on the foundation modulus is considered to be insignificant relative to the other uncertainties involved in determining rock foundation properties, and it is not measured.

(5) To account for the uncertainties, a lower and upper bound for the foundation modulus should be used for each rock type modeled in the structural analysis.

3-3. Loads

a. General. In the design of concrete gravity dams, it is essential to determine the loads required in the stability and stress analysis. The following forces may affect the design:

(1) Dead load.

(2) Headwater and tailwater pressures.

(3) Uplift.

(4) Temperature.

(5) Earth and silt pressures.

(6) Ice pressure.

(7) Earthquake forces.

(8) Wind pressure.

(9) Subatmospheric pressure.

(10) Wave pressure.

(11) Reaction of foundation.

b. Dead load. The unit weight of concrete generally should be assumed to be 150 pounds per cubic foot until an exact unit weight is determined from the concrete materials investigation. In the computation of the dead load, relatively small voids such as galleries are normally not deducted except in low dams, where such voids could create an appreciable effect upon the stability of the structure. The dead loads considered should include the weight of concrete, superimposed backfill, and appurtenances such as gates and bridges.

c. Headwater and tailwater.

(1) General. The headwater and tailwater loadings acting on a dam are determined from the hydrology, meteorology, and reservoir regulation studies. The frequency of the different pool levels will need to be determined to assess which will be used in the various load conditions analyzed in the design.

(2) Headwater.

(a) The hydrostatic pressure against the dam is a function of the water depth times the unit weight of water. The unit weight should be taken at 62.5 pounds per cubic foot, even though the weight varies slightly with temperature.

(b) In some cases the jet of water on an overflow section will exert pressure on the structure. Normally such forces should be neglected in the stability analysis except as noted in paragraph 3-3*i*.

(3) Tailwater.

(a) For design of nonoverflow sections. The hydrostatic pressure on the downstream face of a nonoverflow section due to tailwater shall be determined using the full tailwater depth.

(b) For design of overflow sections. Tailwater pressure must be adjusted for retrogression when the flow conditions result in a significant hydraulic jump in the downstream channel, i.e. spillway flow plunging deep into tailwater. The forces acting on the downstream face of overflow sections due to tailwater may fluctuate significantly as energy is dissipated in the stilling basin. Therefore, these forces must be conservatively estimated when used as a stabilizing force in a stability analysis. Studies have shown that the influence of tailwater retrogression can reduce the effective tailwater depth used to calculate pressures and forces to as little as 60 percent of the full tailwater depth. The amount of reduction in the effective depth used to determine tailwater forces is a function of the degree of submergence of the crest of the structure and the backwater conditions in the downstream channel. For new designs, Chapter 7 of EM 1110-2-1603 provides guidance in the calculation of hydraulic pressure distributions in spillway flip buckets due to tailwater conditions.

(c) Tailwater submergence. When tailwater conditions significantly reduce or eliminate the hydraulic jump in the spillway basin, tailwater retrogression can be neglected and 100 percent of the tailwater depth can be used to determine tailwater forces.

(d) Uplift due to tailwater. Full tailwater depth will be used to calculate uplift pressures at the toe of the structure in all cases, regardless of the overflow conditions.

d. Uplift. Uplift pressure resulting from headwater and tailwater exists through cross sections within the dam, at the interface between the dam and the foundation, and within the foundation below the base. This pressure is present within the cracks, pores, joints, and seams in the concrete and foundation material. Uplift pressure is an active force that must be included in the stability and stress analysis to ensure structural adequacy. These pressures vary with time and are related to boundary conditions and the permeability of the material. Uplift pressures are assumed to be unchanged by earthquake loads.

(1) Along the base.

(a) General. The uplift pressure will be considered as acting over 100 percent of the base. A hydraulic gradient between the upper and lower pool is developed between the heel and toe of the dam. The pressure distribution along the base and in the foundation is dependent on the effectiveness of drains and grout curtain, where applicable, and geologic features such as rock permeability, seams, jointing, and faulting. The uplift pressure at any point under the structure will be tailwater pressure plus the pressure measured as an ordinate from tailwater to the hydraulic gradient between upper and lower pool.

(b) Without drains. Where there have not been any provisions provided for uplift reduction, the hydraulic gradient will be assumed to vary, as a straight line, from headwater at the heel to zero or tailwater at the toe. Determination of uplift, at any point on or below the foundation, is demonstrated in Figure 3-1.

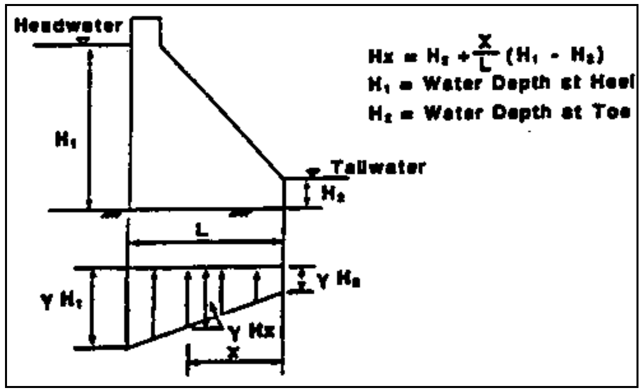

Figure 3-1. Uplift distribution without foundation drainage

(c) With drains. Uplift pressures at the base or below the foundation can be reduced by installing foundation drains. The effectiveness of the drainage system will depend on depth, size, and spacing of the drains; the character of the foundation; and the facility with which the drains can be maintained. This effectiveness will be assumed to vary from 25 to 50 percent, and the design memoranda should contain supporting data for the assumption used. If foundation testing and flow analysis provide supporting justification, the drain effectiveness can be increased to a maximum of 67 percent with approval from CECW-ED. This criterion deviation will depend on the pool level operation plan instrumentation to verify and evaluate uplift assumptions and an adequate drain maintenance program. Along the base, the uplift pressure will vary linearly from the undrained pressure head at the heel, to the reduced pressure head at the line of drains, to the undrained pressure head at the toe, as shown in Figure 3-2. Where the line of drains intersects the foundation within a distance of 5 percent of the reservoir depth from the upstream face, the uplift may be assumed to vary as a single straight line, which would be the case if the drains were exactly at the heel. This condition is illustrated in Figure 3-3. If the drainage gallery is above tailwater elevation, the pressure of the line of drains should be determined as though the tailwater level is equal to the gallery elevation.

(d) Grout curtain. For drainage to be controlled economically, retarding of flow to the drains from the upstream head is mandatory. This may be accomplished by a zone of grouting (curtain) or by the natural imperviousness of the foundation. A grouted zone (curtain) should be used wherever the foundation is amenable to grouting. Grout holes shall be oriented to intercept the maximum number of rock fractures to maximize its effectiveness. Under average conditions, the depth of the grout zone should be two-thirds to three-fourths of the headwater-tailwater differential and should be supplemented by foundation drain holes with a depth of at least

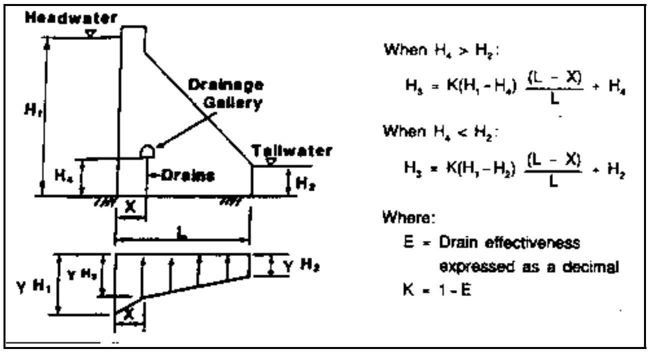

Figure 3-2. Uplift distribution with drainage gallery

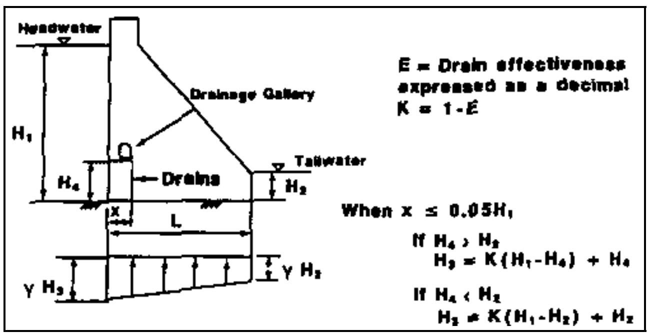

Figure 3-3. Uplift distribution with foundation drains near upstream face

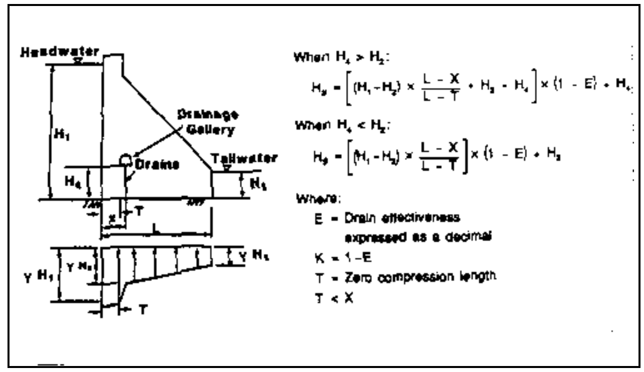

Figure 3-4. Uplift distribution cracked base with drainage, zero compression zone not extending beyond drains

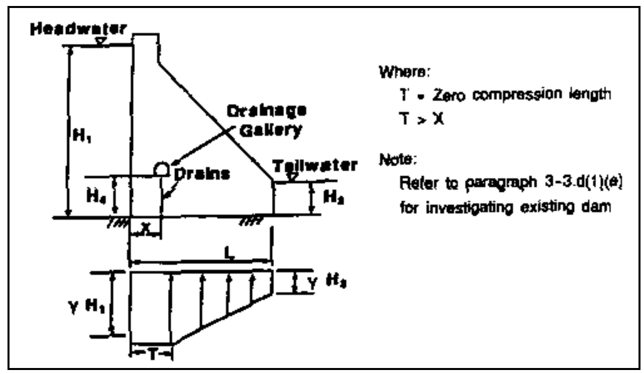

Figure 3-5. Uplift distribution cracked base with drainage, zero compression zone extending beyond drains

two-thirds that of the grout zone (curtain). Where the foundation is sufficiently impervious to retard the flow and where grouting would be impractical, an artificial cutoff is usually unnecessary. Drains, however, should be provided to relieve the uplift pressures that would build up over a period of time in a relatively impervious medium. In a relatively impervious foundation, drain spacing will be closer than in a relatively permeable foundation.

(e) Zero compression zones. Uplift on any portion of any foundation plane not in compression shall be 100 percent of the hydrostatic head of the adjacent face, except where tension is the result of instantaneous loading resulting from earthquake forces. When the zero compression zone does not extend beyond the location of the drains, the uplift will be as shown in Figure 3-4. For the condition where the zero compression zone extends beyond the drains, drain effectiveness shall not be considered. This uplift condition is shown in Figure 3-5. When an existing dam is being investigated, the design office should submit a request to CECW-ED for a deviation if expensive remedial measures are required to satisfy this loading assumption.

(2) Within dam.

(a) Conventional concrete. Uplift within the body of a conventional concrete-gravity dam shall be assumed to vary linearly from 50 percent of maximum headwater at the upstream face to 50 percent of tailwater, or zero, as the case may be, at the downstream face. This simplification is based on the relative impermeability of intact concrete which precludes the buildup of internal pore pressures. Cracking at the upstream face of an existing dam or weak horizontal construction joints in the body of the dam may affect this assumption. In these cases, uplift along these discontinuities should be determined as described in paragraph 3-3.*d*(1) above.

(b) RCC concrete. The determination of the percent uplift will depend on the mix permeability, lift joint treatment, the placements, techniques specified for minimizing segregation within the mixture, compaction methods, and the treatment for watertightness at the upstream and downstream faces. A porous upstream face and lift joints in conjunction with an impermeable downstream face may result in a pressure gradient through a cross section of the dam considerably greater than that outlined above for conventional concrete. Construction of a test section during the design phase (in accordance with EM 1110-2-2006, Roller Compacted Concrete) shall be used as a means of determining the permeability and, thereby, the exact uplift force for use by the designer.

(3) In the foundation. Sliding stability must be considered along seams or faults in the foundation. Material in these seams or faults may be gouge or other heavily sheared rock, or highly altered rock with low shear resistance. In some cases, the material in these zones is porous and subject to high uplift pressures upon reservoir filling. Before stability analyses are performed, engineering geologists must provide information regarding potential failure planes within the foundation. This includes the location of zones of low shear resistance, the strength of material within these zones, assumed potential failure planes, and maximum uplift pressures that can develop along the failure planes. Although there are no prescribed uplift pressure diagrams that will cover all foundation failure plane conditions, some of the most common assumptions made are illustrated in Figures 3-6 and 3-7. These diagrams assume a uniform head loss along the failure surface from point "A" to tailwater, and assume that the foundation drains penetrate the failure plane and are effective in reducing uplift on that plane. If there is concern that the drains may be ineffective or partially effective in reducing uplift along the failure plane, then uplift distribution as represented by the dashed line in Figures 3-6 and 3-7 should be considered for stability computations. Dangerous uplift pressures can develop along foundation seams or faults if the material in the seams or faults is pervious and the pervious zone is intercepted by the base of the dam or by an impervious fault. These conditions are described in Casagrande (1961) and illustrated by Figures 3-8 and 3-9. Every effort is made to grout pervious zones within the foundation prior to constructing the dam. In cases where grouting is impractical or ineffective, uplift pressure can be reduced to safe levels through proper drainage of the pervious zone. However, in those circumstances where the drains do not penetrate the pervious zone or where drainage is only partially effective, the uplift conditions shown in Figures 3-8 and 3-9 are possible.

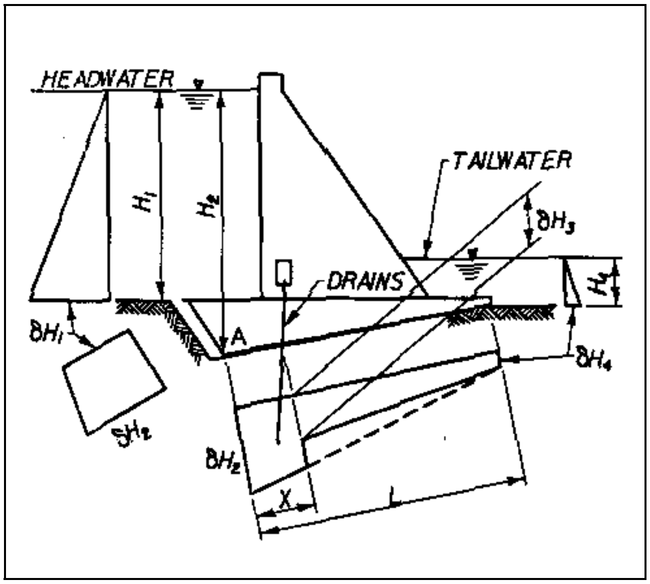

Figure 3-6. Uplift pressure diagram. Dashed line represents uplift distribution to be considered for stability computations

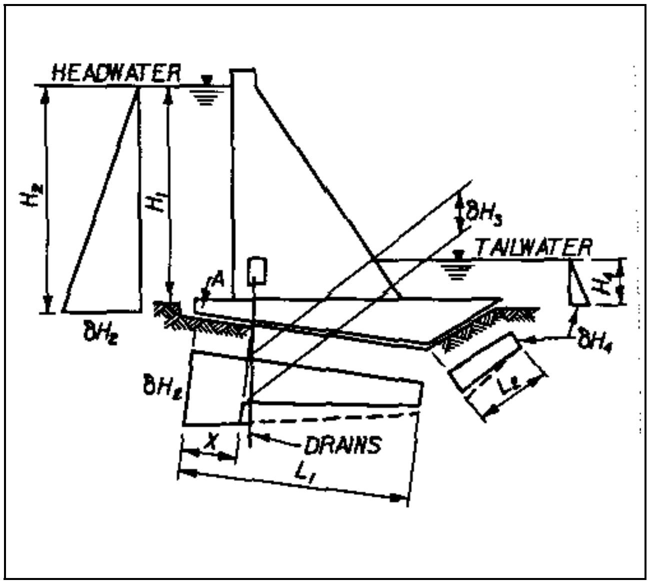

Figure 3-7. Dashed line in uplift pressure diagram represents uplift distribution to be considered for stability computations

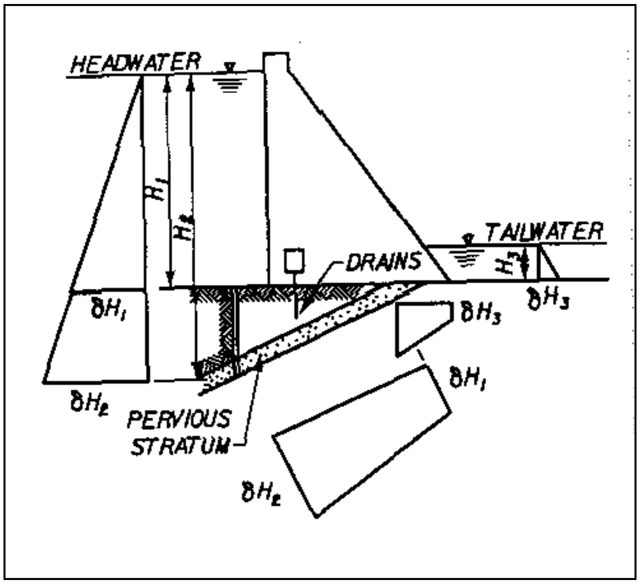

Figure 3-8. Development of dangerous uplift pressure along foundation seams or faults

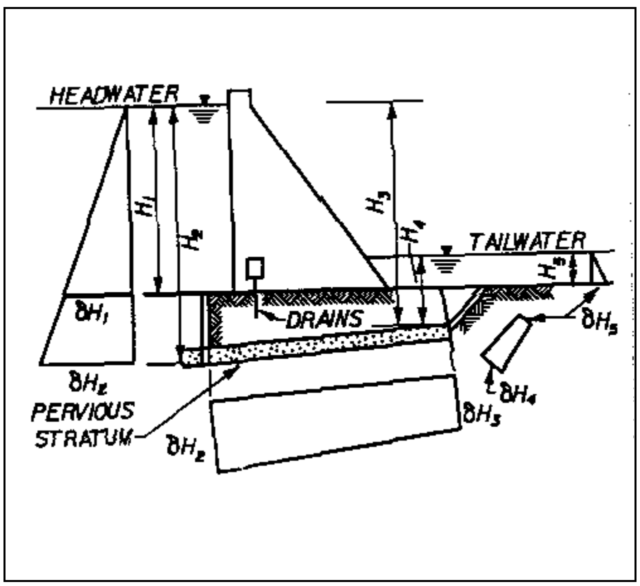

Figure 3-9. Effect along foundation seams or faults if material is pervious and pervious zone is intercepted by base of dam or by impervious fault

e. Temperature.

(1) A major concern in concrete dam construction is the control of cracking resulting from temperature change. During the hydration process, the temperature rises because of the hydration of cement. The edges of the monolith release heat faster than the interior; thus the core will be in compression and the edges in tension. When the strength of the concrete is exceeded, cracks will appear on the surface. When the monolith starts cooling, the contraction of the concrete is restrained by the foundation or concrete layers that have already cooled and hardened. Again, if this tensile strain exceeds the capacity of the concrete, cracks will propagate completely through the monolith. The principal concerns with cracking are that it affects the watertightness, durability, appearance, and stresses throughout the structure and may lead to undesirable crack propagation that impairs structural safety.

(2) In conventional concrete dams, various techniques have been developed to reduce the potential for temperature cracking (ACI 224R-80). Besides contraction joints, these include temperature control measures during construction, cements for limiting heat of hydration, and mix designs with increased tensile strain capacity.

(3) If an RCC dam is built without vertical contraction joints, additional internal restraints are present. Thermal loads combined with dead loads and reservoir loads could create tensile strains in the longitudinal axis sufficient to cause transverse cracks within the dam.

f. Earth and silt. Earth pressures against the dam may occur where backfill is deposited in the foundation excavation and where embankment fills abut and wrap around concrete monoliths. The fill material may or may not be submerged. Silt pressures are considered in the design if suspended sediment measurements indicate that such pressures are expected. Whether the lateral earth pressures will be in an active or an at-rest state is determined by the resulting structure lateral deformation. Methods for computing the Earth's pressures are discussed in EM 1110-2-2502, Retaining and Flood Walls.

g. Ice pressure. Ice pressure is of less importance in the design of a gravity dam than in the design of gates and other appurtenances for the dam. Ice damage to the gates is quite common while there is no known instance of any serious ice damage occurring to the dam. For the purpose of design, a unit pressure of not more than 5,000 pounds per square foot should be applied to the contact surface of the structure. For dams in this country, the ice thickness normally will not exceed 2 feet. Climatology studies will determine whether an allowance for ice pressure is appropriate. Further discussion on types of ice/structure interaction and methods for computing ice forces is provided in EM 1110-2-1612, Ice Engineering.

h. Earthquake.

(1) General.

(a) The earthquake loadings used in the design of concrete gravity dams are based on design earthquakes and site-specific motions determined from seismological evaluation. As a minimum, a seismological evaluation should be performed on all projects located in seismic zones 2, 3, and 4. Seismic zone maps of the United States and Territories and guidance for seismic evaluation of new and existing projects during various levels of design documents are provided in ER 1110-2-1806, Earthquake Design and Analysis for Corps of Engineers Projects.

(b) The seismic coefficient method of analysis should be used in determining the resultant location and sliding stability of dams. Guidance for performing the stability analysis is provided in Chapter 4. In strong seismicity areas, a dynamic seismic analysis is required for the internal stress analysis. The criteria and guidance required in the dynamic stress analysis are given in Chapter 5.

(c) Earthquake loadings should be checked for horizontal earthquake acceleration and, if included in the stress analysis, vertical acceleration. While an earthquake acceleration might take place in any direction, the analysis should be performed for the most unfavorable direction.

(2) Seismic coefficient. The seismic coefficient method of analysis is commonly known as the pseudo-static analysis. Earthquake loading is treated as an inertial force applied statically to the structure. The loadings are of two types: inertia force due to the horizontal acceleration of the dam and hydrodynamic forces resulting from the reaction of the reservoir water against the dam (see Figure 3-10). The magnitude of the inertia forces is computed by the principle of mass times the earthquake acceleration. Inertia forces are assumed to act through the center of gravity of the section or element. The seismic coefficient is a ratio of the earthquake acceleration to gravity; it is a dimensionless unit, and in no case can it be related directly to acceleration from a strong motion instrument. The coefficients used are considered to be the same for the foundation and are uniform for the total height of the dam. Seismic coefficients used in design are based on the seismic zones given in ER 1110-2-1806.

(a) Inertia of concrete for horizontal earthquake acceleration. The force required to accelerate the concrete mass of the dam is determined from the equation:

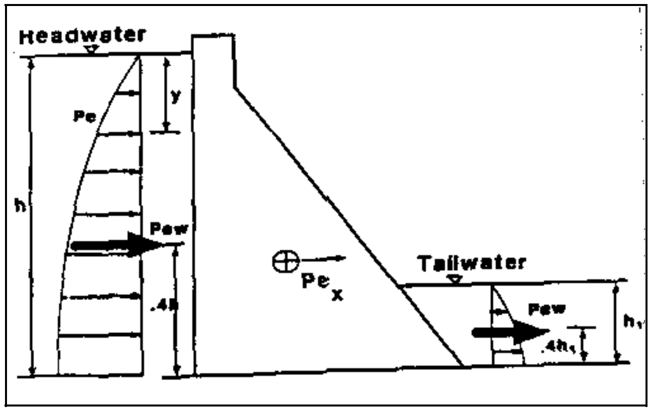

Figure 3-10. Seismically loaded gravity dam, nonoverflow monolith

$$Pe_x = Ma_x = \frac{W}{g} \alpha g = W\alpha \tag{3-2}$$

where

Pe_x = horizontal earthquake force

M = mass of dam

a_x = horizontal earthquake acceleration = g

W = weight of dam

g = acceleration of gravity

α = seismic coefficient

(b) Inertia of reservoir for horizontal earthquake acceleration. The inertia of the reservoir water induces an increased or decreased pressure on the dam concurrently with concrete inertia forces. Figure 3-10 shows the pressures and forces due to earthquake by the seismic coefficient method. This force may be computed by means of the Westergaard formula using the parabolic approximation:

$$Pew = \frac{2}{3}\, Ce\, (\alpha)\, y\, (\sqrt{hy}\,) \tag{3-3}$$

where

Pew = additional total water load down to depth y (kips)

Ce = factor depending principally on depth of water and the earthquake vibration period, t_e, in seconds

h = total height of reservoir (feet)

Westergaard's approximate equation for *Ce*, which is sufficiently accurate for all usual conditions, in pound-second feet units is:

$$Ce = \frac{51}{\sqrt{1 - 0.72\left(\frac{h}{1{,}000\ t_e}\right)^2}} \tag{3-4}$$

where t_e is the period of vibration.

(3) Dynamic loads. The first step in determining earthquake induced loading involves a geological and seismological investigation of the damsite. The objectives of the investigation are to establish the controlling maximum credible earthquake (MCE) and operating basis earthquake (OBE) and the corresponding ground motions for each, and to assess the possibility of earthquake-produced foundation dislocation at the site. The MCE and OBE are defined in Chapter 5. The ground motions are characterized by the site-dependent design response spectra and, when necessary in the analysis, acceleration-time records. The dynamic method of analysis determines the structural response using either a response spectrum or acceleration-time records for the dynamic input.

(a) Site-specific design response spectra. A response spectrum is a plot of the maximum values of acceleration, velocity, and/or displacement of an infinite series of single-degree-of-freedom systems subjected to an earthquake. The maximum response values are expressed as a function of natural period for a given damping value. The site-specific response spectra is developed statistically from response spectra of strong motion records of earthquakes that have similar source and propagation path properties or from the controlling earthquakes and that were recorded on a similar foundation. Application of the response spectra in dam design is described in Chapter 5.

(b) Acceleration--time records. Accelerograms, used for input for the dynamic analysis, provide a simulation of the actual response of the structure to the given seismic ground motion through time. The acceleration-time records should be compatible with the design response spectrum.

i. Subatmospheric pressure. At the hydrostatic head for which the crest profile is designed, the theoretical pressures along the downstream face of an ogee spillway crest approach atmospheric pressure. For heads higher than the design head, subatmospheric pressures are obtained along the spillway. When spillway profiles are designed for heads appreciably less than the probable maximum that could be obtained, the magnitude of these pressures should be determined and considered in the stability analysis. Methods and discussions covering the determination of these pressures are presented in EM 1110-2-1603, Hydraulic Design of Spillways.

j. Wave pressure. While wave pressures are of more importance in their effect upon gates and appurtenances, they may, in some instances, have an appreciable effect upon the dam proper. The height of waves, runup, and wind setup are usually important factors in determining the required freeboard of any dam. Wave dimensions and forces depend on the extent of water surface or fetch, the wind velocity and duration, and other factors. Information relating to waves and wave pressures are presented in the Coastal Engineering Research Center's *Shore Protection Manual* (SPM), Vol II (SPM 1984).

k. Reaction of foundations. In general, the resultant of all horizontal and vertical forces including uplift must be balanced by an equal and opposite reaction at the foundation consisting of the normal and tangential components. For the dam to be in static equilibrium, the location of this reaction is such that the summation of forces and moments are equal to zero. The distribution of the normal component is assumed as linear, with a knowledge that the elastic and plastic properties of the foundation material and concrete affect the actual distribution.

(1) The problem of determining the actual distribution is complicated by the tangential reaction, internal stress relations, and other theoretical considerations. Moreover, variations of foundation materials with depth, cracks, and fissures that interrupt the tensile and shearing resistance of the foundation also make the problem more complex.

(2) For overflow sections, the base width is generally determined by projecting the spillway slope to the foundation line, and all concrete downstream from this line is disregarded. If a vertical longitudinal joint is not provided at this point, the mass of concrete downstream from the theoretical toe must be investigated for internal stresses.

(3) The unit uplift pressure should be added to the computed unit foundation reaction to determine the maximum unit foundation pressure at any point.

(4) Internal stresses and foundation pressures should be computed with and without uplift to determine the maximum condition.